ARTICLE 4

PROCEDURES FOR
MOMENT RESISTING SYSTEMS

4.0 INTRODUCTION

Moment frames develop their resistance to lateral forces through the flexural strength and continuity of beam and column elements. Moment frames may be classified as special, intermediate, and ordinary frames.

For evaluations using these regulations, it is not necessary to determine the type of frame in the building. The issues are addressed by appropriate acceptance criteria in the specified procedures. For determination of element capacities, see Article 2, Section 2.4.9.

4.1 FRAMES WITH INFILL WALLS

4.1.1 INTERFERING WALLS: All infill walls placed in moment frames are isolated from structural elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is an inappropriate connection of the wall to the frame. Evaluate the relative strength and stiffness of the walls and frames, considering the nature and size of the joint or connection between the wall and the frame. If the strength of the walls is not commensurate with the stiffness, the building should be treated as Type 7 or Type 10 (Article 2, Section 2.2.3 "Common Building Types), a frame with infill walls. If the infill walls do not extend the full story height and are not properly isolated from the frame columns, evaluate the column shear demand and capacity, based on a column height equal to the clear distance from the top of wall to the bottom of the slab or beam above, amplifying the design forces in the short column by $C_d/2$, but not less than 1.5. The shear demand need not exceed the flexural capacity of the column, based on a column height equal to the clear distance from the top of wall to the bottom of the slab or beam above.

4.2 STEEL MOMENT FRAMES

Welded steel moment frames may be subject to detailed frame joint evaluation requirements, as outlined in this section. The purpose of this joint evaluation is to determine if the building has experienced joint damage in strong ground shaking.

4.2.0.1 Preliminary Screening

All welded steel moment frame structures shall undergo a detailed frame joint evaluation if the building is located at a site that has experienced the following:

1) An earthquake of magnitude greater than or equal to 6.5, which produced ground motion in excess of 0.20g; or

2) An earthquake that generated ground motion in excess of 0.30g.

The ground motion estimates shall be based on actual instrumental recordings in the vicinity of the building. When such ground motion records are not available, ground motion estimates may be based on empirical or analytical techniques. All ground motion estimates shall reflect the site-specific soil conditions.

4.2.0.2 Additional Indicators

A detailed frame joint evaluation of the building shall be performed if any of the following apply:

1) Significant structural damage is observed in one or more welded steel moment frame structures located within 1 kilometer of the building, on sites with similar, or more firm soil properties;

2) An earthquake having a magnitude of 6.5 or greater, the structure is within 5 kilometers of the trace of a surface rupture or within the vertical projection of the rupture area when no surface rupture has occurred;

3) Significant architectural or structural damage has been observed in the building following an earthquake; or

4) Entry to the building has been limited by the building official because of earthquake damage, regardless of the type or nature of the damage.

4.2.0.3 Connection Inspections

Detailed frame joint evaluations shall be performed in accordance with the procedures in the *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, FEMA 267, August 1995.

4.2.1 DRIFT CHECK: The building satisfies the Quick Check of the frame drift.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check drift using the procedures in Section 2.4.7.1 against the prescribed limit. If the drift exceeds the limiting drift at any story level, the structure shall be evaluated with full-frame analysis using the anticipated distribution of lateral forces to the moment resisting frames and including *P*-delta effects. Check the other statements using the demand from this analysis.

4.2.2 COMPACT MEMBERS: All moment frame elements meet the compact section requirements of the basic AISC documents.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the member capacities. Check member capacities, using member demands obtained from a frame analysis. Calculate member capacities using appropriate criteria for noncompact sections. Check the member capacities using appropriate *R* values (e.g., noncompact members require use of the *R* value for ordinary frames).

4.2.3 BEAM PENETRATIONS: All openings in frame-beam webs have a depth less than 1/4 of the beam depth and are located in the center half of the beams.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the shear capacity of the beam. Check that the shear capacity of the beam is sufficient to develop the flexural plastic hinge. If the shear capacity is insufficient to develop the flexural capacity of the member, use of the *R* value for ordinary frames.

4.2.4 MOMENT CONNECTIONS: All beam-column connections in the lateral-force-resisting moment frame have full-penetration flange welds and a bolted or welded web connection.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connection. Check the connection on the basis of its strength. Check the member capacities using appropriate *R* values. Connections that do not develop the flexural capacity of the member require use of the *R* value for ordinary frames.

4.2.5 COLUMN SPLICES: All column splice details of the moment resisting frames include connection of both flanges and the web.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the bolts or welds in the connection. Check the adequacy of the splice connection for all gravity and seismic loads. Amplify the seismic load for partial-penetration welded splices by the factor $C_d/2$.

4.2.6 JOINT WEBS: All web thicknesses within joints of moment resisting frames meet AISC criteria for web shear.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the web. Calculate the joint shear capacity using formulas given in the AISC provisions and compare it to the demand from an equivalent lateral force analysis or the average column shear, V_c, calculated for the Quick Check for drift.

4.2.7 GIRDER FLANGE CONTINUITY PLATES: There are girder flange continuity plates at joints.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the joint. Check joints without such plates using AISC provisions, using the *R* value for ordinary frames.

4.2.8 STRONG COLUMN/WEAK BEAM: At least one half of the joints in each story are strong column/weak beam (33 percent on every line of moment frame). Roof joints need not be considered.

The deficiency is excessive ductility demand and displacement in a single story. Compare beam and column moment capacities including the effect of axial force. The evaluator may consider this condition mitigated if the joints in the building meet the provisions of Section 2710(g)5 of the 1992 Edition of Part 2, Title 24. Conforming buildings which do not meet those provisions shall be placed in SPC 4.

4.2.9 OUT-OF-PLANE BRACING: Beam-column joints are braced out-of-plane.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the stability of the beam-column joint. Verify the joint bracing by visual observation.

4.2.10 PRE-NORTHRIDGE EARTHQUAKE WELDED MOMENT FRAME JOINTS: Welded steel moment frame beam-column joints are designed and constructed in accordance with recommendations in FEMA 267, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, August 1995.

For buildings constructed under permit issued after October 25, 1994, the evaluator may consider this condition as mitigated. The deficiency is in the ductility of the beam-column joint. The following procedures shall be used for categorizing buildings with welded steel moment frame joints:

Procedure for conforming buildings:

Conforming buildings located in Seismic Zone 4, within a zone designated as being potentially subject to near field effects in strong ground shaking, shall be placed in SPC 3. All other conforming buildings shall be placed in SPC 4.

Procedure for nonconforming buildings:

Nonconforming buildings shall be placed in SPC 2.

4.3 CONCRETE MOMENT FRAMES

The details covered in evaluation statements 4.3.4 through 4.3.14 will be found in frames that have been designed and detailed for ductile behavior. If any one detail is not present, the frames are not considered to meet life-safety goals, and nonconforming buildings shall be placed in SPC 1. For conforming buildings see the appropriate evaluation statement. For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the building may assume "true" responses to all evaluation statements in Section 4.3.

4.3.1 SHEARING STRESS CHECK: The building satisfies the Quick Check of the average shearing stress in the columns.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Perform a quick estimation of the average shearing stress in the columns according to the procedure specified in Sec. 2.4.7.2. If the average column shear stress is greater than 60 psi, a more detailed evaluation of the structure shall be performed. This evaluation shall employ a more accurate estimation of the level and distribution of the lateral loads; use the procedures outlined in Sec. 2.4.

4.3.2 DRIFT CHECK: The building satisfies the Quick Check of story drift.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check drift using the procedures in Section 2.4.7.1 against the prescribed limit. If the drift exceeds the limiting drift at any story level, the structure shall be evaluated with full-frame analysis using the anticipated distribution of lateral forces to the moment resisting frames and including P-delta effects as found in Section 2.4.1. Check the other statements using the demand from this analysis.

4.3.3 PRESTRESSED FRAME ELEMENTS: The lateral-load-resisting frames do not include any prestressed or post-tensioned elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the frames during inelastic straining. Check the capacity of the members and joints using all of the mild steel reinforcing that is available and bonded prestressing when appropriate. The R value used for evaluation shall reflect the ductility and damping of the system. Where better information is not available, multiply the R value selected on the basis of mild reinforcement by 0.75 to account for the effect of prestressing.

4.3.4 JOINT ECCENTRICITY: There are no eccentricities larger than 20 percent of the smallest column plan dimension between girder and column centerlines.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the frame, either the members or the joints or both. Evaluate the frames considering the additional shear stresses caused by the joint torsion.

4.3.5 NO SHEAR FAILURES: The shear capacity of frame members is greater than the moment capacity.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is inadequate shear capacity in the columns or beams. Compare V_e with the member shear capacity, ϕV_n, calculated in accordance with ACI 318 Appendix. The ratio $V_e/\phi V_n$ shall be less than or equal to 1.0.

4.3.6 STRONG COLUMN/WEAK BEAM: The moment capacity of the columns is greater than that of the beams.

The deficiency is in column capacity. Compare the sum of the beam moment capacities to that of the column capacities. Include the participation of the slab in the beam capacities. The moment capacity to be compared is the plastic moment, M_{pr}. The ratio of the sum of the M_{pr} for the columns to the sum of the M_{pr} for the beams is required to be not less than 1.2. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.7 STIRRUP AND TIE HOOKS: The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more.

The deficiency is in the shear resistance and confinement of the member. Determine if beam stirrups and column ties are appropriately anchored into member cores with hooks of 135 degrees or more. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.8 COLUMN-TIE SPACING: Frame columns have ties spaced at d/4 or less throughout their length and at 8 d_b or less at all potential plastic hinge regions.

The deficiency is in the shear capacity of the column. Report this condition as a deficiency. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.9 COLUMN-BAR SPLICES: All column bar lap splice lengths are greater than 35 d_b long and are enclosed by ties spaced at 8 d_b or less.

The deficiency is in the strength and ductility of the column. Compare the splice length provided with that required by Sec. 12.2 and 12.15 of the ACI 318 provisions. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.10 BEAM BARS: At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25 percent of the steel provided at the joints for either positive or negative moment is continuous throughout the members.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam. Determine if the required beam bars are present. For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary.

4.3.11 BEAM-BAR SPLICES: The lap splices for longitudinal beam reinforcing are located within the center half of the member lengths and not in the vicinity of potential plastic hinges.

The deficiency is in the strength and ductility of the beam. Determine if the beam bar splices are detailed and located such that the yield capacity of the beam can be developed. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

4.3.12 STIRRUP SPACING: All beams have stirrups spaced at d/2 or less throughout their length and at 8 d_b or less at potential hinge locations.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam. Determine if the stirrups meet the specified spacing requirements, such that the yield capacity of the beam can be developed.

4.3.13 BEAM TRUSS BARS: Bent-up longitudinal steel is not used for shear reinforcement.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam. Determine if bent-up shear reinforcement is present. If present, check the shear capacity of the element ignoring the effects of the bent-up longitudinal bars.

4.3.14 JOINT REINFORCING: Column ties extend at their typical spacing through all beam-column joints at exterior columns.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength and ductility of the beam-column joint. Calculate the joint capacity, V_e, and the joint shear, V_j. The joint shear is calculated at a horizontal section at mid-height of the joint. The horizontal shear at the critical section is obtained from summation of horizontal forces in a free-body diagram of the upper half of the joint as $V_j = (T_l + T_r) - V_e$ where T_l and T_r, the forces in the flexural tensile reinforcement in the beams on the left and right sides of the joint, respectively, are calculated assuming a steel stress equal to 1.25 f_y.. See Figure 4.3.14 for computation of V_e. The ratio V_j/V_e shall be less than or equal to 1. Conforming buildings which do not meet this criteria shall be placed in SPC 4.

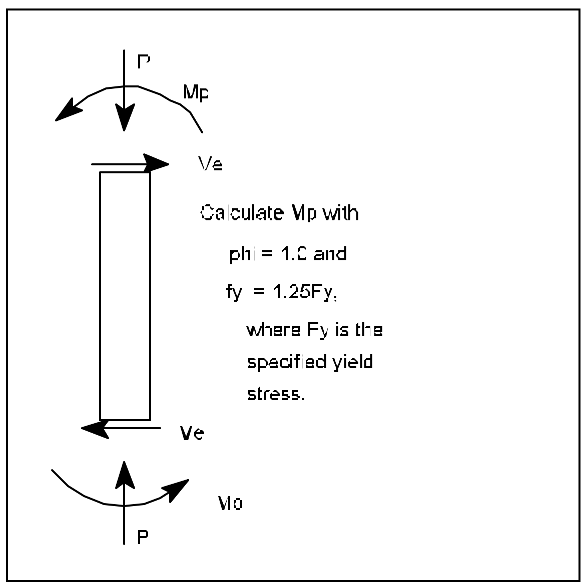

FIGURE 4.3.14 M_{pr} and V_e.

4.3.15 FLAT SLAB FRAMES: The system is not a frame consisting of columns and a flat slab/plate without beams.

For buildings designed and constructed in accordance with the 1989 or later editions of Part 2, Title 24, the evaluator may consider this condition as mitigated, and no calculations are necessary. Perform a detailed analysis, or assign the building to SPC 1.

4.4 PRECAST CONCRETE MOMENT FRAMES

4.4.1 PRECAST FRAMES: The lateral loads are not resisted by precast concrete frame elements.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connections. Check the adequacy of the precast frames. Where lateral movement will cause strength capacities to be first exceeded at connections, use $R = C_d = 1.5$ unless there is information on connection behavior that justifies higher values. Where all yielding occurs within members, use the R value for the appropriate cast-in-place frame.

4.4.2 PRECAST CONNECTIONS: For buildings with concrete shear walls, the connection between precast frame elements such as chords, ties, and collectors in the lateral-force-resisting system can develop the capacity of the connected members.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. The deficiency is in the strength of the connections. Analyze the connections. Determine where connection failures would be brittle (e.g., pull-out of an embedded item would occur before yield of a mild steel element). Analyze structure for stability assuming that these brittle connections have failed or are not capable of transmitting forces, or check such connections for seismic force amplified by factor $C_d/2$, but not less than 1.5. For shear capacity, refer to Sec. 4.3. For flexure, find the path of forces from the element through the connection into the other element.

4.5 FRAMES NOT PART OF THE LATERAL-FORCE-RESISTING SYSTEM

This section deals with frames that were not designed to be part of the lateral-force-resisting system. These are basic structural frames of steel or concrete that are designed for gravity loads with shear walls, bracing, or moment frames providing the resistance to lateral forces.

If the primary lateral force resisting system consist of concrete walls (infilled in steel frames or monolithic in concrete frames), the building shall be treated as a concrete shear wall building (Type 6) with the frame columns as boundary elements. If the walls are masonry infills, the frames shall be treated as steel or concrete frames with infill walls of masonry (Type 7 or 10). Buildings with steel braces shall be treated as braced frame systems (Type 4). The principal deficiency identified in this section is loss of vertical-load-carrying capacity due to excessive deformations.

The analysis must include the deformations imposed by the infill walls, and the consequences of the failure of such walls.

4.5.1 COMPLETE FRAMES: The steel or concrete frames form a complete vertical load carrying system.

For conforming buildings, the evaluator may consider this condition as mitigated, and no calculations are necessary. Check the shear walls or braced frames, including the effects of all dead and live loads and noting that the R values for buildings without a complete vertical load carrying space frame are different from those for complete frame buildings. For wall systems, the frame is consider to be incomplete if the beams end at the edge of a shear wall that has no boundary columns or, if there are such columns, the beams do not continue across in the plane of the wall. For chevron braced frame systems, the frame is consider incomplete if the beam in the brace frame cannot carry the design dead and live loads without the presence of the braces.